CHAPTER 5
ANALYSIS PROCEDURES

5-1. General.

a. Introduction. This chapter defines four basic analytical procedures; however, only the first three procedures are prescribed by this document. The first two procedures are linear elastic, and the latter two procedures are nonlinear. Limitations on the use of linear elastic static procedures are indicated in Paragraph 5-2b, and conditions when nonlinear procedures are required are provided in Paragraph 5-4b. The procedures are discussed in the following paragraphs in order of increasing rigor and complexity. Advantages, disadvantages, and limitations are indicated for each procedure. Paragraph 4-11 and Table 4-4 prescribe the minimum analytical procedure for each performance objective for the various seismic use groups. The prescribed minimum procedure is intended to apply to structures of average complexity for each performance objective. Unusual, or more complex, structures may require more complex or rigorous analytical procedures than the prescribed minimum.

b. Mathematical Modeling.

(1) Basic assumptions. In general, a building should be modeled, analyzed, and evaluated as a three-dimensional assembly of elements and components. Three-dimensional mathematical models shall be used for analysis and evaluation of buildings with plan irregularity. Two-dimensional modeling, analysis, and evaluation of buildings with stiff or rigid diaphragms is acceptable if torsional effects are either sufficiently small to be ignored or indirectly captured. Vertical lines of seismic framing in buildings with flexible diaphragms may be individually modeled, analyzed, and evaluated as two-dimensional assemblies of components and elements, or a three-dimensional model may be used with the diaphragms modeled as flexible elements. Connection modeling is not required for linear analysis. Explicit modeling of a connection is required for nonlinear analysis if the connection is weaker than the connected components, and/or the flexibility of the connection results in a significant increase in the relative deformation between the connected components.

(2) Horizontal torsion. The effects of horizontal torsion must be considered for buildings with diaphragms capable of resisting such torsion. The total torsional moment at a given floor level shall be set equal to the sum of the following two torsional moments:

- The actual torsion; that is, the moment, M_t, resulting from the eccentricity between the centers of mass at all floors above and including the given floor, and the center of rigidity of the vertical seismic elements in the story below the given floor, and

- The accidental torsion; that is, an accidental torsional moment, M_{ta}, produced by horizontal offset in the center of mass, at all floors above and including the given floor, equal to a minimum of 5 percent of the horizontal dimension at the given floor level measured perpendicular to the direction of the applied load.

For buildings in Seismic Design Categories C, D, E, and F, where torsional irregularity exists as defined in Table 5.2.3.2 of FEMA 302, the effects of the irregularity shall be accounted for by multiplying the sum of M_t and M_{ta} at each level by a torsional amplification factor, A_x, determined from:

$$A_x = \left\{ \frac{d_{\max}}{1.2\, d_{\text{avg}}} \right\}^2 \qquad (5\text{-}1)$$

where:

$d_{\max}$ = maximum displacement at Level x

d_{avg} = average of the displacements at the extreme points of the building at Level x.

The torsional amplification factor, A_x is not required to exceed 3.0.

5-2. Linear Elastic Static Procedure.

a. General. This procedure, also known as the "Equivalent Lateral Force (ELF) Procedure," will be the procedure most widely employed for one-story buildings, and can be utilized for all regular buildings of two to six stories, and is the preferred procedure for structures of wood frame or light metal frame construction. The required calculations are relatively simple and can be performed by hand, although a number of computer programs are available to facilitate the analysis. The results of the linear static analysis procedure can be very inaccurate when applied to buildings with highly irregular structural systems, unless the building is capable of responding to the design earthquake(s) in a nearly elastic manner. Therefore, linear static analysis procedures should not be used for highly irregular buildings, except wood frame structures.

b. Limitations on Use of the Procedure. The linear elastic static procedure may be used unless one or more of the following conditions apply, in which case the linear elastic dynamic procedure, described in Paragraph 5-3, shall be used:

- The building height exceeds 100 feet.

- The ratio of the building's horizontal dimensions at any story to the corresponding dimensions at an adjacent story exceeds 1.4 (excluding penthouse).

- The building is found to have a severe torsional stiffness irregularity in any story. A severe torsional stiffness irregularity may be deemed to exist in a story if the diaphragm above the story is not flexible, and the results of the analysis indicate that the drift along any side of the structure is more than 150 percent of the average story drift.

- The building is found to have a severe vertical mass or stiffness irregularity. A severe vertical mass or stiffness irregularity may be deemed to exist when the average drift in any story (except penthouses) exceeds that of the story above or below by more than 150 percent.

- The building has a non-orthogonal lateral-force-resisting system.

c. Implementation of the Procedure shall be in accordance with the provisions of Section 5.3 of FEMA 302, with exceptions or modifications as noted in the following paragraphs.

(1) Performance Objective 1A. In accordance with Section 5.3 of FEMA 302, except that I = 1.0. This is the prescribed analysis in FEMA 302 for standard occupancy structures, and is prescribed in this document as a preliminary analysis for all seismic use groups to satisfy the Life Safety Performance Objective.

(2) Performance Objective 2A, 2B, and 3B. In accordance with Section 5.3 of FEMA 302, except that R = 1.0, I = 1.0, and the base shear is modified to represent the pseudo-lateral load described in paragraph 6-3a(2). The m modification factors used in these analyses are defined in Paragraph 6-3a.

Exception: Buildings with enhanced performance objectives in Seismic Design Categories A and B may be analyzed by the ELF procedure, or the modal analysis procedure described in the following paragraph, with the appropriate ground motions, the R factors from Table7-1, and the applicable I factor from FEMA 302.

5-3. Linear Elastic Dynamic Procedure.

This procedure, also known as the "Modal Analysis Procedure," shall be performed in accordance with the requirements of Section 5.4 of FEMA 302, with the exceptions noted in Paragraph 5-2c for use with the various performance objectives. For most moment frame systems, the contribution of panel zone deformations to overall story drift may be assumed to be adequately represented by the use of centerline-to-centerline dimensions in the mathematical model. This analytical procedure is considered acceptable for all structures and all performance objectives designed in accordance with this document, except for the structures incorporating the use of a supplemental energy dissipation system and some types of base isolation systems. For specific analysis procedures applicable to those structures, refer to Chapter 8. The ELF procedure described in Paragraph 5-2 may be more appropriate for some regular or rigid (one or two-story) structures. Unusual or complex structures in Seismic Use Groups II and III, with the characteristics described in Paragraph 5-4b, may require a nonlinear elastic static procedure for confirmation of the enhanced Performance Objectives 2A, 2B, and 3B.

5-4. Nonlinear Static Procedure.

a. General. Nonlinear procedures directly account for the redistribution of forces and deformations that occur in a structure as it undergoes inelastic response. Consequently, they are generally capable of providing a more accurate estimate of the demands produced in a structure than either of the linear procedures. However, the nonlinear static procedure is not able to predict accurately the higher mode response of flexible structures and a nonlinear dynamic procedure should be considered for tall buildings (i.e. in excess of six stories) or buildings with significant vertical irregularities.

b. When Nonlinear Procedures are Required. In order to determine whether a building may be analyzed with sufficient accuracy by linear procedures, it is necessary to perform a linear analysis and then examine the results to determine the magnitude and distribution of inelastic demands on the various components of the primary lateral-force-resisting elements. The magnitude and distribution of inelastic demands are indicated by demand-capacity ratios (DCRs). DCRs for existing and new building components shall be computed in accordance with the equation:

$$\mathrm{DCR} = \frac{Q_{UD}}{Q_{CE}} \qquad (5\text{-}2)$$

where:

Q_{UD} = the combined effect of gravity loads and earthquake loads

Q_{CE} = the expected strength of the component or element at the deformation level under consideration for deformation-controlled actions.

DCRs should be calculated for each controlling action (such as axial force, moment, and shear) of each component. If all of the computed controlling DCRs for a component is less than or equal to 1.0, then the component is expected to respond elastically to the earthquake ground shaking being evaluated. If one or more of the computed DCRs for a component is greater than 1.0, then the component is expected to respond inelastically to the earthquake ground shaking. The largest DCR calculated for a given component defines the critical action for the component, i.e., the mode in which the deformation-controlled component will first yield, or fail in the case of a brittle force-controlled component. This DCR is termed the critical component DCR. If an element is composed of multiple components, then the components with the largest computed DCR is the critical component for the element, i.e., this will be the first component in the element to yield, or fail. The largest DCR for any component in an element at a particular story is termed the critical element DCR at that story. If the DCRs computed for all of the critical actions (axial force, moment, shear) of all of the components (such as beams, columns, wall piers, braces, and connections) of the primary elements are less than 2.0, then linear analysis procedures are applicable, regardless of considerations of regularity. If some computed DCRs exceed 2.0, then linear methods should not be used if any of the following apply:

- There is an in-plane discontinuity in any primary element of the lateral-force-resisting system. In-plane discontinuities occur whenever a lateral-force-resisting element is present in one story, but does not continue, or is offset, in the story immediately below. Figure 5-1 indicates such a condition.

- There is an out-of-plane discontinuity in any primary element of the lateral-force-resisting system. An out-of-plane discontinuity exists when an element in one story is offset relative to the continuation of that element in an adjacent story, as indicated in Figure 5-2.

- There is a severe weak story irregularity present at any story in any direction of the building. A severe weak story irregularity may be deemed to

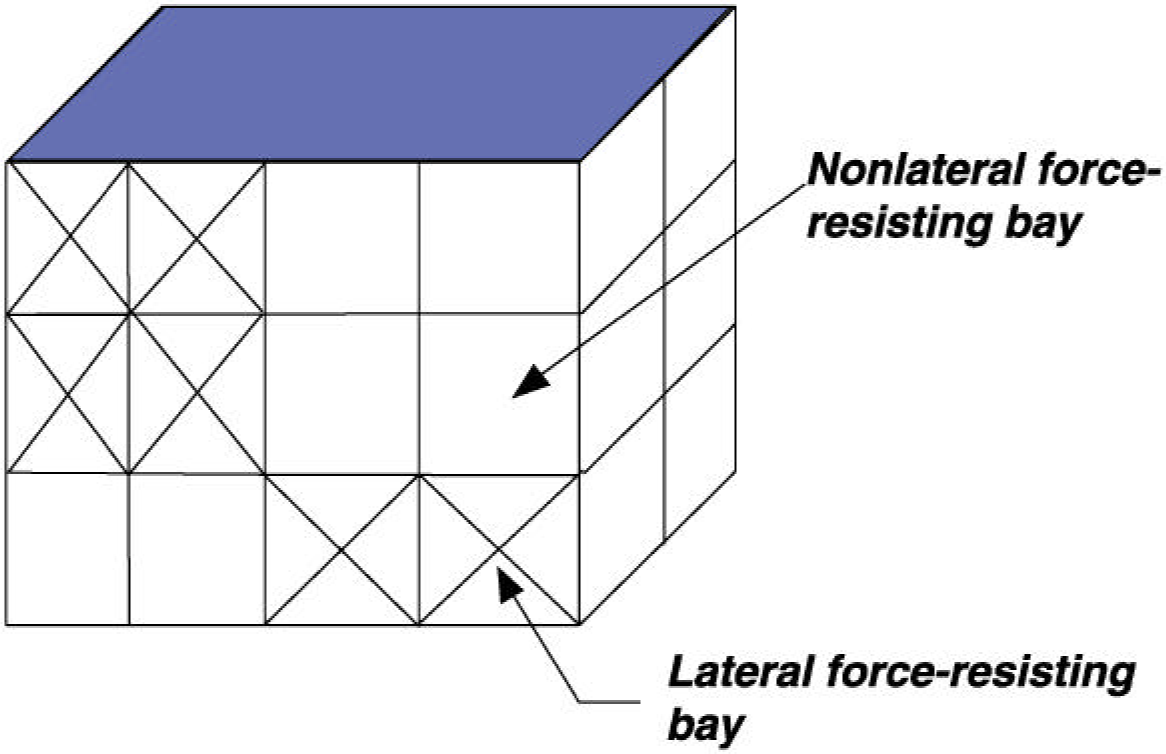

Figure 5-1: In-Plane Discontinuity in Lateral System

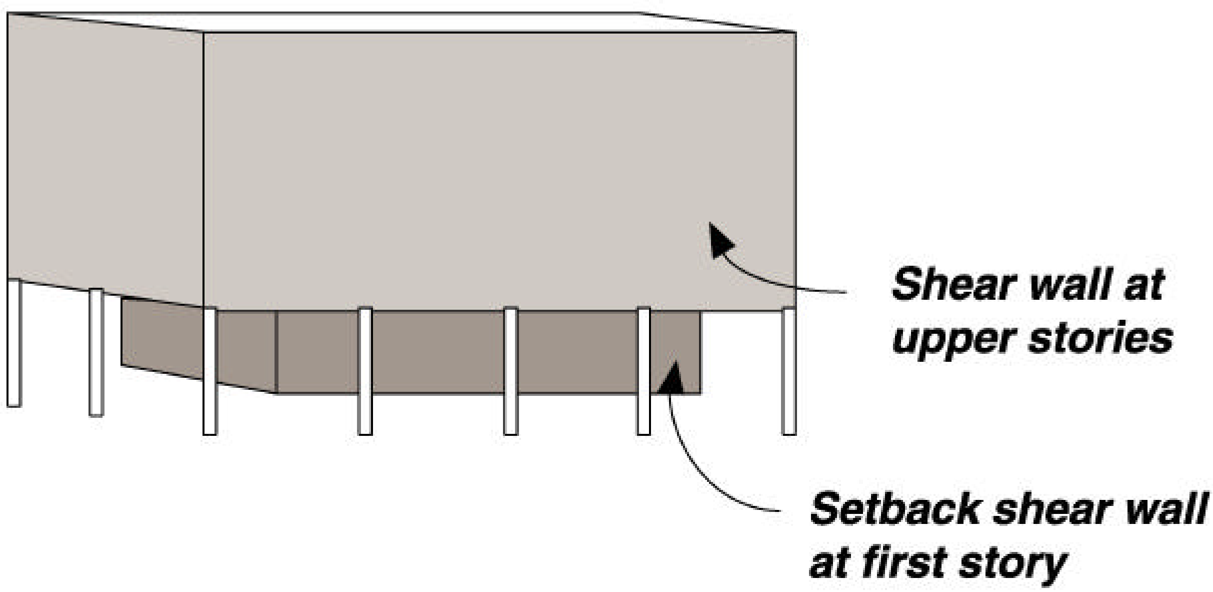

Figure 5-2: Typical Building with Out-of-Plane Offset Irregularity

exist if the ratio of the average shear DCR for any story to that for an adjacent story in the same direction exceeds 125 percent. The average DCR for a story may be calculated by the equation:

$$\overline{\mathrm{DCR}} = \frac{\sum_{1}^{n} \mathrm{DCR}_i V_i}{\sum_{1}^{n} V_i} \qquad (5\text{-}3)$$

where:

DCR = the average DCR for the story

DCR_I = the critical action DCR for element *i*

V_I = the total calculated lateral shear force in an element *i* due to earthquake response, assuming that the structure remains elastic

n = the total number of elements in the story.

- There is a severe torsional strength irregularity present in any story. A severe torsional strength irregularity may be deemed to exist in a story when the diaphragm above the story is not flexible, and the ratio of the critical element DCRs for primary elements on one side of the center of resistance in a given direction for a story, to those on the other side of the center of resistance for the story, exceeds 1.5.

c. Limitations on Use of the Procedure. The nonlinear static procedure may be used for any structure and for any performance objective, with the following exceptions and limitations:

- The use of the nonlinear static procedure in this document is required for those structures in Seismic Use Groups II and III with the structural characteristics described in Paragraph 5-4b, unless specific instructions to the contrary are received from the cognizant design authority.

- The procedure is not recommended for use with wood-frame structures.

- The procedure should not be used for structures in which higher-mode effects are significant unless a linear elastic dynamic procedure is also performed. To determine if higher-mode effects are significant, a modal response spectrum analysis shall be performed for the structure using sufficient modes to capture 90 percent mass participation, and a second modal response spectrum analysis shall be performed considering only the first mode participation. Higher-mode effects shall be considered significant if the shear in any story calculated from the analysis with 90 percent mass participation exceeds 130 percent of the corresponding story shear from the analysis considering only the first mode response. A linear elastic dynamic procedure may be performed to supplement the nonlinear static procedure for structures with significant higher-mode effects, in which case the acceptance criteria values for deformation-controlled actions (*m* values) in Chapter 7 may be increased by a factor of 1.3.

d. *Basis of the Procedure*. Under the Nonlinear Static Procedure, a model directly incorporating inelastic material response is displaced to a target displacement, and resulting internal deformations and forces are determined. The nonlinear-load-deformation characteristics of individual components and elements of the building are modeled directly. The mathematical model of the building is subjected to monotonically increasing lateral forces or displacements until either a target displacement is exceeded, or the building collapses. The target displacement is intended to represent the maximum displacement likely to be experienced during the design earthquake. The target displacement may be calculated by any procedure that accounts for the effects of nonlinear response on displacement amplitude; one rational procedure is presented in Paragraph 5-4f, and further described in Paragraph 4-8b(3) and Table 4-7. Because the mathematical model accounts directly for effects of material inelastic response, the calculated internal forces will be reasonable approximations of those expected during the design earthquake. The target displacement calculated using Equation 5-5 may be unconservative if the strength ratio of Equation 5-6 exceeds five, or if the building is located in the near field of the causative fault. Results of the Nonlinear Static Procedure are to be checked using the applicable acceptance criteria prescribed in Chapter 6, and provided in Chapter 7. Calculated displacements and internal forces are to be compared directly with the allowable values.

e. *Modeling and Analysis Criteria.*

(1) General. In this document, the Nonlinear Static Procedure involves the monotonic application of lateral forces or displacements to a nonlinear mathematical model of a building until the displacement of the control node in the mathematical model exceeds a target displacement. For buildings that are not symmetric about a plane perpendicular to the applied lateral loads, the lateral loads must be applied in both the positive and negative directions, and the maximum forces and deformations used for design. The relation between base shear force and lateral displacement of the target node shall be established for control node displacements ranging between zero and 150 percent of the target displacement, $*_t$, given by Equation 5-5. Acceptance criteria shall be based on those forces and deformations (in components and elements) corresponding to a minimum horizontal displacement of the control node equal to $*_t$. Gravity loads shall be applied to appropriate elements and components of the mathematical model during the nonlinear analysis. The loads and load combination presented in ASCE 7, as appropriate, shall be used to represent the gravity loads. The analysis model shall be discretized in sufficient detail to represent adequately the load-deformation response of each component along its length. Particular attention should be paid to identifying locations of inelastic action along the length of a component, as well as at its ends.

(2) Control node. The procedure requires definition of the control node in a building. This document considers the control node to be the center of mass at the roof of a building. The top of a penthouse should not be considered as the roof. The

displacement of the control node is compared with the target displacement, a displacement that characterizes the effects of earthquake shaking.

(3) Lateral load patterns. Lateral loads shall be applied to the building in profiles that approximately bound the likely distribution of inertia forces in an earthquake. For three-dimensional analysis, the horizontal distribution should simulate the distribution of inertia forces in the plane of each floor diaphragm. For both two- and three-dimensional analysis, the vertical distributions of lateral load shall be selected from one of the following two options:

- A lateral-load pattern represented by values of C_{VX} given in Equation 5.3.4-1 of FEMA 302, which may be used if more than 75 percent of the total mass participates in the fundamental mode in the direction under consideration.

- A lateral-load pattern proportional to the story inertia forces consistent with the story shear distribution calculated by combination of modal responses using (1) response spectrum analysis of the building including a sufficient number of modes to capture 90 percent of the total mass; and (2) the appropriate ground-motion spectrum.

(4) Period determination. The effective fundamental period T_e in the direction under consideration shall be calculated using the force-displacement relationship of the Nonlinear Procedure. The nonlinear relation between base shear and displacement of the target node shall be replaced with a bilinear relation to estimate the effective lateral stiffness, K_e, and the yield strength, V_y, of the building as indicated in Figure 5-3. The effective lateral stiffness shall be taken as the secant stiffness calculated at a base shear force equal to 60 percent of the yield strength. The effective fundamental period T_e shall be calculated as:

$$T_e = T_i \sqrt{\frac{K_i}{K_e}} \qquad (5\text{-}4)$$

where:

T_i = elastic fundamental period (in seconds) in the direction under consideration calculated by elastic dynamic analysis

K_i = elastic lateral stiffness of the building in the direction under consideration

K_e = effective lateral stiffness of the building in the direction under consideration.

See Figure 5-3 for further information.

(5) Analysis of three-dimensional models. Static lateral forces shall be imposed on the three-dimensional mathematical model corresponding to the mass distribution at each floor level. The effects of accidental torsion shall be considered. Independent analysis along each principal axis of the three-dimensional mathematical model is permitted unless multi-directional evaluation is required, as prescribed in Section 5.2.6.3.1 of FEMA 302.

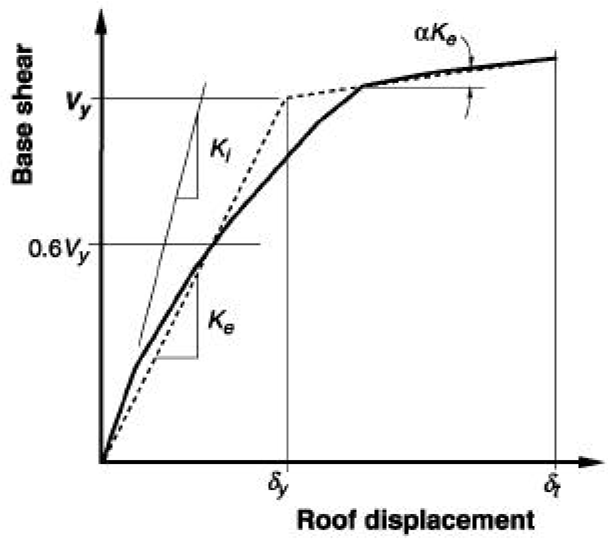

Figure 5-3: Calculation of Effective Stiffness, K_e

(6) Analysis of two-dimensional models. Mathematical models describing the framing along each axis (axis 1 and axis 2) of the structure shall be developed for two-dimensional analysis. The effects of horizontal torsion shall be considered. If multidirectional excitation effects are to be considered, component deformation demands and actions shall be computed for the following cases: 100 percent of the target displacement along axis 1, and 30 percent of the target displacement along axis 2; and 30 percent of the target displacement along axis 1, and 100 percent of the target displacement along axis 2.

f. Determination of Actions and Deformations.

(1) Pushover curve. The general procedure for the development of the load/displacement or pushover curve is as follows.

(a) An elastic structural model is developed that includes all components having significant contributions to the weight, strength, stiffness, and/or stability of the structure, and whose behavior is important in satisfying the desired level of seismic performance. The structure is loaded with gravity loads in the same load combination(s) as used in the linear procedures before proceeding with the application of lateral loads.

(b) The structure is subjected to a set of lateral loads, using one of the load patterns (distributions) described in Paragraph 5-4e(3).

(c) The intensity of the lateral load is increased until the weakest component reaches a deformation at which its stiffness changes significantly (usually the yield load or member strength). The stiffness properties of this "yielded" component in the structural model are modified to reflect post-yield behavior, and the modified structure is subjected to an increase in lateral-loads (load control) or displacements (displacement control), using the same shape of the lateral-load distribution, or an updated shape to reflect the revised fundamental mode shape. Modification of component behavior may be in one of the following forms:

- Placing a hinge where a flexural element has reached its bending strength; this may be at the end of a beam, column, or base of a shear wall.

- Eliminating the shear stiffness of a shear wall that has reached its shear strength in a particular story.

- Eliminating a bracing element that has buckled and whose post-buckling strength decreases at a rapid rate.

- Modifying stiffness properties if an element is capable of carrying more loads with a reduced stiffness.

(d) Step (c) is repeated as more and more components reach their strength. Note that although the intensity of loading is gradually increasing, the load pattern usually remains the same for all stages of the "yielded" structure, unless the user decides on the application of an adaptive load pattern. At each stage, internal forces and elastic and plastic deformations of all components are calculated.

(e) The loading process is continued until unacceptable performance is detected or a roof displacement is obtained that is larger than the maximum displacement specified in Paragraph 5-4e(1). Unacceptable performance may be defined as excessive drift of the building, or the undesirable response or failure of critical components or elements.

(f) The forces and deformations from all previous loading stages are accumulated to obtain the total forces and deformations (elastic and plastic) of all components at all loading stages.

Note: Steps (c) through (f) can be performed systematically with a nonlinear computer analysis program using an event-by-event strategy, or an incremental analysis with predetermined displacement increments in which iterations are performed to balance internal forces.

(g) The displacement of the control node versus first story (base) shear at various loading stages is plotted as a representative nonlinear response diagram of the structure. The changes in slope of this curve are indicative of the yielding of various components.

(h) The control node displacement versus base shear curve is used to estimate the target displacement, as described in the following paragraph. Note that this step may require iteration if the yield strength and stiffnesses of the simplified bilinear relation are sensitive to the target displacement.

(i) Once the target displacement is known, the accumulated forces and deformations at this displacement of the control node should be used to evaluate the performance of components and elements.

1. For deformation-controlled actions (e.g., flexure in beams), the deformation demands are compared with the maximum permissible values given in Chapter 7.

2. For force-controlled actions (e.g., shear in beams), the lower-bound strength capacity is compared with the force demand. Capacities are given in Chapters 7 through 10.

(j) If either (a) the force demand in force-controlled actions, components, or elements, or (b) the deformation demand in deformation-controlled actions, components, or elements, exceeds permissible values by more than 10 percent, then the action, component, or element is deemed to violate the performance criterion.

(k) When the demand exceeds the permissible capacity of the components or elements as described in Step (j) above, the mathematical model of the building shall be redesigned to provide additional strength and/or rigidity to the deficient components or elements, and the pushover procedure shall be repeated, as necessary. Similarly, if the evaluation indicates that a number of components or elements are overdesigned by a factor of 10 percent or more, the mathematical model shall be redesigned, and the pushover procedure repeated unless the overdesign can be justified as being cost-effective or otherwise beneficial.

(2) Target displacement. The target displacement $\ast_t$ for a building with rigid diaphragms at each floor level shall be estimated using an established procedure that accounts for the likely nonlinear response of the building. Actions and deformations corresponding to the control node displacement equaling or exceeding the target displacement shall be used for component checking in Chapter 7. The procedure for evaluating the target displacement is given by the following equation:

$$\ast_{\mathrm{t}} = C_0\, C_1\, C_2\, C_3\, S_a\, \frac{T_e^{\,2}}{4\Pi^2} \qquad (5\text{-}5)$$

where:

T_e = effective fundamental period of the building in the direction under consideration

C_0 = modification for C_0 can be calculated using one of the following:

- the first modal participation factor at the level of the control node

- the modal participation factor at the level of the control node calculated using a shape vector corresponding to the deflected shape of the building at the target displacement
- the appropriate value from Table 5-1.

C_1 = modification factor to relate expected maximum inelastic displacement to displacements calculated for linear elastic response.

= 1.0 for $T_e \geq T_s$

= $[1.0 + (R - 1)\, T_s/T_e]/R$ for $T_e < T_s$

but need not exceed:

$C_1 = 1.5$ for $T_e < 0.10$ sec.

C_1 may be interpreted between $T_e = 0.10$ sec and $T_e = T_s$.

T_s = a characteristic period of the response spectrum, defined as the period associated with the transition from the constant acceleration segment of the spectrum to the constant velocity segment of the spectrum.

R = ratio of elastic strength demand to calculated yield strength coefficient. See below for additional information.

C_2 = modification factor to represent the effect of hysteresis shape on the maximum displacement response. Values for C_2 are established in Table 5-2.

C_3 = Modification factor to represent increased displacement due to dynamic P-) effects. For buildings with positive post-yield stiffness, C_3 shall be set equal to 1.0. For buildings with negative post-yield stiffness, values of C_3 shall be calculated using Equation 5-7.

S_a = response spectrum acceleration, at the effective fundamental period and damping ratio of the building in the direction under consideration.

The strength ratio R shall be calculated as:

Number of Stories	Modification Factor[1]
1	1
2	1.2
3	1.3
5	1.4
10+	1.5

1. Linear interpolation should be used to calculate intermediate values.

Table 5-1: Values for Modification Factor C_0

	T = 0.1 second		T >= T_s second	
Performance Level	**Framing Type1[1]**	**Framing Type 2[2]**	**Framing Type 1[1]**	**Framing Type 2[2]**
Immediate Occupancy	1.0	1.0	1.0	1.0
Life Safety	1.3	1.0	1.1	1.0
Collapse Prevention	1.5	1.0	1.2	1.0

1. Structures in which more than 30% of the story shear at any level is resisted by components or elements whose strength and stiffness may deteriorate during the design earthquake. Such elements and components include: ordinary moment-resisting frames, concentrically-braced frames, frames with partially-restrained connections, tension-only braced frames, unreinforced masonry walls, shear-critical wall and piers, or any combination of the above.

2. All frames not assigned to Framing Type 1.

Table 5-2: Values for Modification Factor C_2

$$R = \frac{S_a}{V_y / W} \cdot \frac{1}{C_0} \qquad (5\text{-}6)$$

where S_a and C_0 are defined above, and:

V_y = yield strength calculated using results of Nonlinear Static Procedure, where the nonlinear force-displacement (i.e., base shear force versus control node displacement) curve of building is characterized by a bilinear relation (Figure 5-3).

W = total dead load and anticipated live load.

Coefficient C_3 shall be calculated as follows if the relation between base shear force and control node displacement exhibits negative post-yield stiffness.

$$C_3 = 1.0 + \frac{|a|(R-1)^{3/2}}{T_e} \qquad (5\text{-}7)$$

where R and T_e are defined above, and:

" = ratio of post-yield stiffness to effective elastic stiffness, where the nonlinear force-displacement relation is characterized by a bilinear relation (Figure 5-3).

For a building with flexible diaphragms at each floor level, a target displacement shall be estimated for each line of vertical seismic framing. The target displacements shall be estimated using an established procedure that accounts for the likely nonlinear response of the seismic framing. One procedure for evaluating the target displacement for an individual line of vertical seismic framing is given by Equation 5-5. The fundamental period of each vertical line of seismic framing, for calculation of the target displacement, shall follow the general procedures described for the Nonlinear Static Procedure; masses shall be assigned to each level of the mathematical model on the basis of tributary area. For a building with neither rigid nor flexible diaphragms at each floor level, the target displacement shall be calculated using rational procedures. One acceptable procedure for including the effects of diaphragm flexibility is to multiply the displacement calculated using Equation 5-5 by the ratio of the maximum displacement at any point on the roof, and the displacement of the center of mass of the roof, both calculated by modal analysis of a three-dimensional model of the building using the design response spectrum. The target displacement so calculated shall be no less than that displacement given by Equation 5-5, assuming rigid diaphragms at each floor level. No vertical line of seismic framing shall be evaluated for displacements smaller than the target displacement.

5-5. Nonlinear Dynamic Procedure.

Under the Nonlinear Dynamic Procedure, design seismic forces, and the corresponding internal forces and system displacements are determined using an inelastic response history dynamic analysis. The basis, modeling approaches, and acceptance criteria for the Nonlinear Dynamic Procedure are similar to those of the Nonlinear Static Procedure. The main exception is that the response calculations are carried out using time-history (also known as response-history) analysis. With the Nonlinear Dynamic Procedure, the design displacements are not established using a target displacement, but instead are determined directly through dynamic analysis using ground-motion histories. These analyses are highly sensitive to the modeling assumptions and to the representation of the ground motion. They are not prescribed by this document, and should only be employed by experienced analysts with express authorization of the cognizant design authority.

5-6. Alternative Rational Analyses.

Nothing in this document should be interpreted as preventing the use of any alternative analysis procedure that is rational and based on fundamental principles of engineering mechanics and dynamics. Such alternative analyses should not adopt the acceptance criteria contained in this document without careful review as to their applicability. All projects using alternative rational analysis procedures should be subject to review by an independent third-party professional engineer, approved by the cognizant design authority, with substantial experience in seismic design.